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Abstract

The United States has experienced an increase in the need for structural repair, especially in its public transportation infrastructure. At the same time, societal requirements to keep these structures open have placed pressure on facility engineers to perform rapid retrofits which entail minimal disruption to these systems. This need has brought the use of Fiber Reinforced Polymers (FRP) to the forefront, as its properties have proven to be invaluable in other industries such as boating, aircraft, and recreation. Its high strength-to-weight ratio, non-corrosive nature, relatively simple application techniques, and non-invasive application procedures have moved FRPs to the top of the list compared to its strengthening contemporaries, mainly steel, in terms of structural retrofits.

This paper will provide an introduction to this technology. It will also present a consolidated summary of various research studies completed in this area. It will look at the effects of FRPs in CMU block and concrete structures. Furthermore, this paper will look into how variables such as different configurations of application, the use of different FRP materials, different application methods, etc. affect the efficiency of its strengthening capabilities. Finally, this paper will take a brief look into where the technology of structural FRP strengthening has evolved to today.

As an officer in the US Navy Civil Engineer Corps, this technology is directly applicable to military requirements. From the standpoint of a military engineer, there is an enormous need for structural strengthening during military operations in countries with inferior structural technology. This is, often times, related to troop, material, and equipment movement over dilapidated transportation infrastructure which can not support the movement.

Introduction

From the beginning of time, man has had a burning desire to build structures. These structures have served many purposes ranging from art, to purely functional, to a combination of both. Over the long history of human construction practices, countless developments have been discovered to improve the capabilities, both in terms of processes and materials.

In this day and age, man has, for the most part, settled into using the same construction materials for their structures. Concrete, concrete masonry units (CMU) and steel make up a good majority of the facilities that are erect on the earth today. As these materials are not new, engineers and builders have had time to realize the strengths as well as the weaknesses of these materials over the life of their structures. As the structures of the world get older, the need for structural repair increases. This is due to strength degradations cause by various factors such as environmental effects, poor design and workmanship upon installation, or simply from structures reaching the end of the design life. One can look at reinforced concrete structures as an example.

Concrete is a material that performs extremely well in compression, although very poorly in tension and shear. Structures made of reinforced concrete take advantage of its high compressive strengths while using steel rebar to provide tensile and shear integrity in the sections of the structural member that requires it. However, one of the major issues with reinforced concrete structures is the degradation of the steel rebar. This degradation is mainly caused by corrosion.

Insufficient concrete coverage of the reinforcing steel can leave it vulnerable. Also, settings that present the reinforced concrete structures with high amounts of aggressive agents including environmental factors (i.e. freeze/thaw environments) can lead to cracking of the concrete coverage and subsequent exposure of the reinforcing elements. Whatever the reason may be, this dilapidation of the reinforcing steel has brought out the weaknesses in the concrete and placed the structures at risk. Transportation structures like bridges and overpasses, due to their purpose and locations, are very likely to be exposed to the conditions mentioned above. A study conducted in 2000 states that almost 40% of bridges in the US are structurally deficient or functional obsolete, and the percentage is increasing. (Griffiths, 2000)

At this point, facility managers are faced with retrofitting these structures. Doing nothing can present unacceptable risks to the users of the facility. Alternative retrofit methods (for column to beam joints) were tested in a study by Murat Engindeniz (2005) which included removal and replacement, epoxy repair, steel jacketing, and FRP. Removal and replacement is a relatively significant procedure that entails extensive facility closure which, in most cases, is not feasible due to the societal need of the facility, especially if it is a major transportation route for the area. Also, temporary supports of the members being removed and replaced would be required. Epoxy repair had limited success in restoring the bond of reinforcing steel, and some believe that this method is inadequate and unreliable. Steel jacketing or addition of steel elements is somewhat effective, although it is very labor intensive. There is significant difficulty in handling the steel jackets and members. Furthermore, it continues to have corrosive issues, not to mention the objectionable aesthetics that it possesses. (Engindeniz, 2005)

This common dilemma has lead to much research in the area of structural retrofitting, which has brought rise to a relatively new method that uses Fiber Reinforced Polymers (FRPs).

What are FRPs and why do they Work?

Fiber Reinforced Polymers (FRPs) have been used in other capacities for several years now. Industries such as boating, automotive, aerospace, and recreation, fishing rods, tennis racquets, ski equipment, and golf clubs are a few that have use this material. Fiber reinforced polymers (FRPs) are composite materials consisting of 3 main components:

- fiber reinforcement
- resin
- and fillers

The fibers provide increased stiffness and tensile capacity. The resin acts like a binder in containing the fibers in a firm matrix and offers high compressive strength. These are the 2 most important elements of the FRP. The filler is used mostly as the name implies, to fill the voids and reduce the cost constructing the FRP. FRPs are anisotropic, meaning that its modulus in the transverse direction is different than that in the longitudinal direction (longitudinal modulus is stronger).

The combination of these three components creates a material that has proved to be a feasible response to the dilemma presented above; but WHY? According to Engindeniz, the FRPs show several pluses like "high strength-weight ratio, corrosion resistance, ease of application (including limited disruption to building occupancy), low labor cost, and no significant increase in member size." (Engindeniz, 2005) The downsides mentioned in Engindeniz's study were the need for improved anchoring of the FRP to prevent debonding. Also, the high initial cost was an issue of mention.

To restate the positive points, FRPs are not susceptible to the corrosive issues of steel. Also, like steel, they exhibit high tensile properties. The physical application of FRPs as a structural retrofit material involves a process much less significant and less imposing than the alternative retrofitting methods. In fact, in some cases, structural retrofits can be accomplished without any facility closure at all. The FRP composites sheets are very thin, therefore not requiring much space for use. The final major benefit of FRPs is that it's lightweight. The use of FRPs does not contribute significant dead loads to the structure of which it is reinforcing. In general, the strength-to-weight ratio makes the use of FRPs extremely effective.

The pictures in Figure 1 below show bridge column retrofit using welded steel jackets and FRPs. The steel alternative contributes much more weight to the structure than FRP thus increasing its deal load.

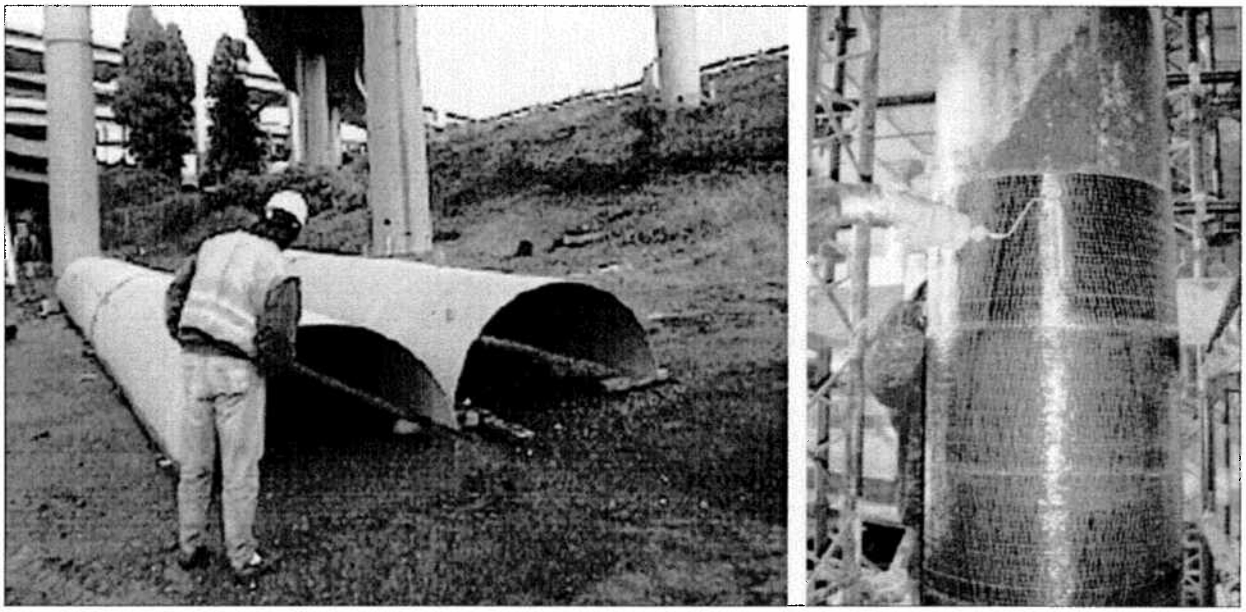


Figure 1: Column Retrofits: Steel and FRP (Kapur; Degussa)

FRP Components

Fibers

The first major constituent of an FRP composite is the fiber itself. In general, the fiber reinforcement occupies approximately 30% - 70% of the FRP material volume (FHWA, 1997). There are 3 common fibers that are used today. They are:

- fiberglass
- aramid
- and carbon

Fiberglass

There are 3 classes of glass fibers: E-glass, S-glass, and C-glass. E-glass is generally designated for electrical use, S-glass for high strength, and C-glass for corrosion resistance. E-glass is the most common glass material for structural reinforcement. As shown in Table 1, glass fiber has the lowest stiffness and strength of the three fibers. It does exhibit ductile properties with a tensile elongation of 2.4%. The strength and modulus of glass fiber can degrade with increased temperature. (FHWA, 1997) Fiberglass is the lowest costing fiber of the three, with a price for .167mm thick sheet costing $7.5/m^2 in 2004 (Xiong, 2004).

Aramid

Aramid fibers consist of aromatic polyamides. They have excellent creep resistance and fatigue. There are two common commercial grades used, Kevlar 29 and Kevlar 49. From Table 1, the stiffness, tensile strength, and elongation are slightly higher than glass. Its cost is more expensive than glass and very similar or a little less expensive than carbon. (FHWA, 1997)

Carbon

Carbons (or graphite) are the strongest of the fibers. From Table 1, carbon fibers are two to three time stiffer than the other two fibers. It has the lowest tensile elongation so low failure strains can be expected. Carbon fibers are the most expensive of the all three fibers, with the price of a .11mm thick sheet costing $35/m^2 in 2004. (Xiong, 2004) This is 5 times the cost of fiberglass that same year. (FHWA, 1997)

Typical Properties	E-glass	Aramid (Kevlar 29)	Carbon
Young's Modulus (GPa)	72	90	230
Tensile Strength (GPa)	1.72	2.27	2.48
Tensile Elongation (%)	2.4	2.8	1.1

Table 1: Fiber Properties (FHWA, 1997)

Resin

Resin is the constituent of the FRP that works as the binder. They are solid at room temperature, and melt upon heating. There are two classes of resins, thermoplastics and thermosets. Since thermoplastics do not cure permanently, they can not be used for structural applications. Thermosets do cure permanently and are very desirable for structural applications. (FHWA, 1997)

Fillers

The cost of resin is not cheap; therefore it would very expensive to completely fill the voids in the fiber sheets completely with resin. For cost efficiency, fillers are used to help occupy the space in and around the fibers. Fillers are also used to improve various characteristics of the FRP. For example, they can be used to improve material properties, aesthetics, manufacturing process, and performance. (FHWA, 1997)

Manufacturing Process

There are three processes for manufacturing FRPs. They are:

- pultrusion
- filament winding
- layup process

The process used for the purpose of structural strengthening is pultrusion. This process involves the pulling of the fibers and mats through a resin bath at elevated temperatures. Upon mixing the fibers and resin, there is a curing period. (FHWA, 1997) Figure 2 provides photos of the pultrusion process of manufacturing FRPs.

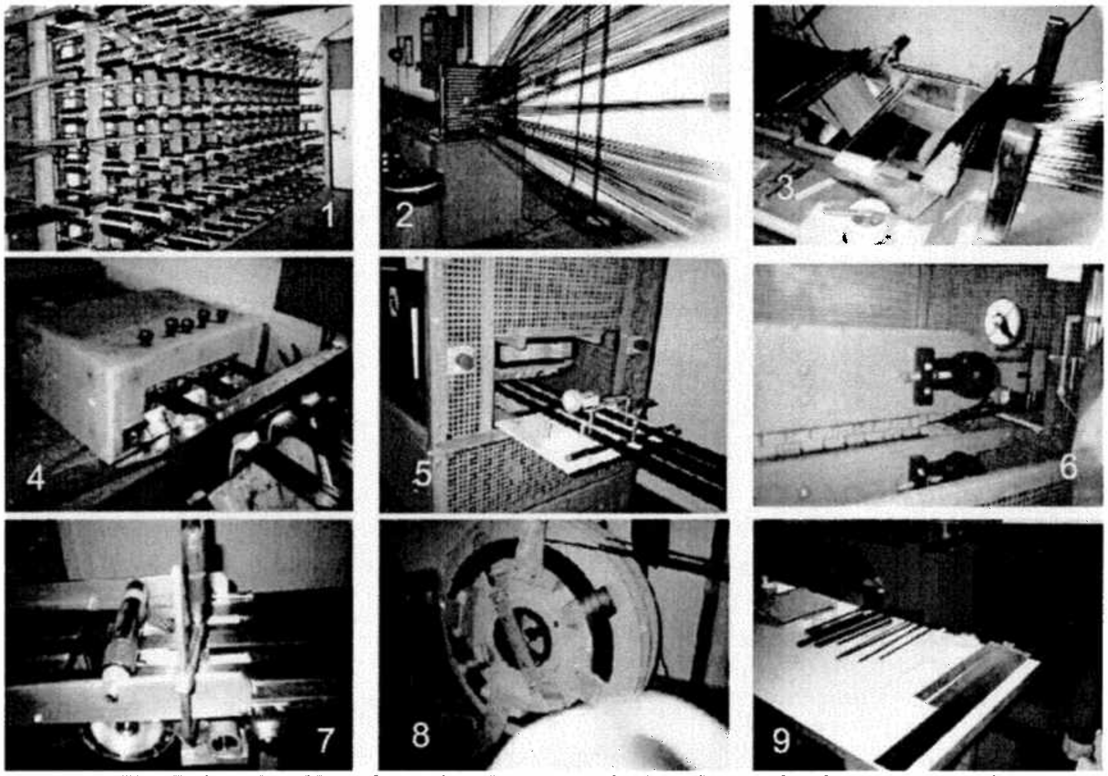


Figure 2: The Pultrusion Manufacturing Process. Carbon fiber is fed from a series of spools (Photo 1) through the spacer screen (Photo 2). Fibers then pass through a resin bath (Photo 3) and between rollers that remove excess resin (Photo 3). The fibers then pass through the laminate strip-shaper (Photo 4), and into the curing oven (Photo 4). Curing is at 750 degrees centigrade and cooling is through baths to room temperature. The cured and cooled laminate then passes through the "Puller" (Photos 5&6), between rollers (Photo 7) and finally, is rolled on the storage spool (Photo 8), or cut to length and stored in another appropriate manner. Various shapes can be manufactured as depicted by photo 9. (FHWA, 1997)

Conventional Application Methods for Structural Retrofit

The conventional method for applying FRPs for structural strengthening is somewhat of a modified, on-site version of the pultrusion method explained above. Not only is the formation of the FRP sheet important, but the bond between the FRP and the construction material is equally important. Therefore, the first part of application is typically the preparation of the construction material surface. This part involves any means necessary (sandblasting, grinding, cleaning, smoothing, etc.) to ensure that there is a good bond between the FRP and the construction material. The next part is typically applying to the material a coat of resin with an instrument like a brush or roller. The fiber sheet is then either directly laid on top of the 1^{st} resin coat, or soaked in the resin and then laid on top of the 1^{st} resin coat. Another resin coat is then applied over the fiber sheet. A period of four plus days is then required to allow the FRP to cure and reach its design strength. (Hamid, 2005; Adhikary, 2004; Xiong, 2004; Masterbuilders, 1998)

FRP Strength with Various Materials and Configurations

As mentioned earlier, FRPs are a great solution in strengthening conventional construction materials. They provide a practical and more effective strengthening solution in comparison to traditional structural renovation techniques. In order to realize the benefits of using FRPs as a material strengthening tool, it is important to understand its behavioral properties in different situations such as with different construction materials and in various application configurations. This section of the paper will look at these situations in detail. It will summarize previous studies conducted on the use of FRPs for material strengthening.

The strengthening characteristics will be examined in terms of different variables. These variables include:

- Performance with different materials such as concrete masonry unit (CMU) blocks and concrete beams
- Performance in different loading configurations
- FRP performance in different strength areas types such as tension and shear
- Performance when the FRPs are applied in different configurations around the structural element
- Performance differences based on how the FRP is physically attached to the structural element
- and performance based on the type of FRP used (carbon, glass, hybrid)

FRP Strengthening on CMU Block Structures

A good percentage of the worlds building inventory is comprised of CMU buildings, many of which have historical value. As this implies, these buildings are somewhat old. They have been subjected to elements and forces that come with an aging building including material wear, various loading conditions, and natural occurrences such as earthquakes and foundation settlements. Since many of these buildings are of historical and architectural significance, limitations for renovation and demolition exist. Solutions for retrofit to meet current building requirements and uses call for the need to strengthen these buildings while maintaining as much of the original structure as possible. Also, CMU buildings are very brittle with low ductility and exhibit very poor performance in merely moderate strength earthquakes.

Previous methods to bring these buildings to standards include the addition of new shear walls or structural frames. The problem with this technique is the impracticalities of putting the theory to work. This method is often very expensive and have several restrictions based on the structure type. Other strengthening methods such as grout injection, steel reinforcement, prestressing, and various surface treatments have been attempted, although, problems arise with these methods with the requirement for considerable disruption of normal building functions. FRPs laminates are a good candidate for such a retrofit given its light weight, small thickness, and relative ease of application. (Hamid, 2005)

Studies have been conducted (Hamid, 2005; Albert, 2001) to determine the change in structural integrity of CMU assemblages reinforced with FRPs. In the study by Hamid et al. (2005), the assemblages were constructed and loaded to mimic the stresses felt by the old type bearing and shear type walls which were typically CMU walls of CMU infill walls. The goal was to simulate all possible plane loading applications.

Assemblage/Loading Configurations

There were three basic types of loading configurations in the study by Hamid (shown in Figure 3). They are:

1. *On/Off axis compressio*n – In this configuration, the in-plane compressive loads are applied to the assemblages in 5 different angles. Two of the scenarios involve on axis loading, which is loading parallel and perpendicular to the bed joints. The other 3 scenarios involve the compressive loading of the assemblage along 30, 60, and 90 degree angles with respect to the bed joints.
2. *Diagonal Tension* – This configuration involves the loading of a square assemblage on two opposite corners to test the diagonal tensile (or shear) strength similar to the loading that occurs in infill walls.
3. *Joint Shear* – This configuration involves the loading of two adjacent horizontal members to test for strengthening in the traditional horizontal shear slip failure mode. To ensure that assemblage fails in shear along the bed joint, the space between the 2 horizontal CMU block members were left unfilled (did not contain any mortar).

Of these loading configurations, the *On/Off axis compression* and the *Joint Shear* will be discussed in this paper.

Figure 3 below is an illustration of the different loading configurations mentioned above in Hamid's study.

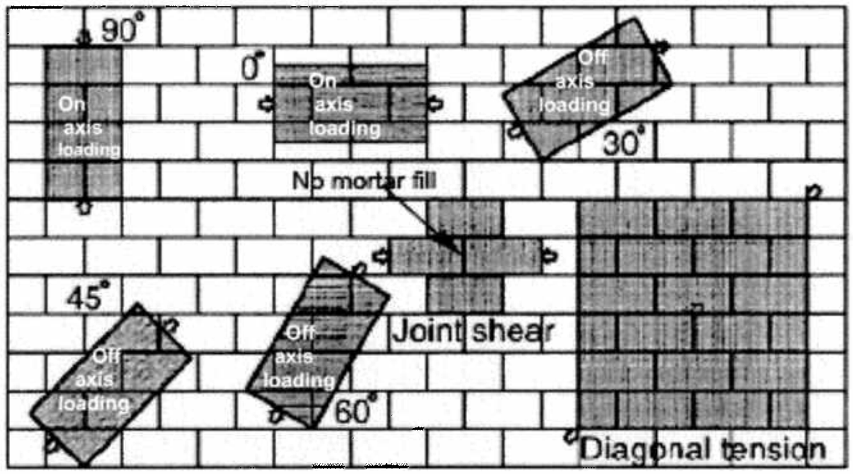


Figure 3: CMU loading configurations (Hamid, 2005)

Three assemblages were constructed for each one of the abovementioned configurations. In addition, three unreinforced (no FRP applied) assemblages were constructed for each of the configurations as control specimens.

During testing, the physical displacement of the specimens at failure was measured using linear variable differential transducers (LVDTs).

<u>FRP Application</u>

The FRP was applied to all of the test assemblages using the conventional method mentioned above. The epoxy is applied to both faces (front and back) of the assemblages using a paint roller. The precut fiber fabric is then place on the wet resin, and then more resin is again applied on top of the fabric. The assemblages were then allowed to cure.

Findings Summary

The performance of the assemblages was described in 3 ways: the strength (compressive or shear), the Young's Modulus, and the failure mode.

0°, 90°, and 60° Assemblages (Compressive Loading)

The 0°, 90° and 60° assemblages can be grouped based on their performance. Each of these assemblages failed due to applied compressive forces. For the 0° and 90° assemblages, they were under pure compression conditions (no applied stresses on the joints due to their on axis loading configurations). The load applied to the 60° assemblage was distributed internally as a combination of compressive and shear loading. It is included in this group because the predominant internal loading in this assemblage was compression, which was evident by the failure mode.

The mode of failure associated with applied compression was vertical splitting through the separation of the internal webbing of the CMU block as displayed in Figure 4.

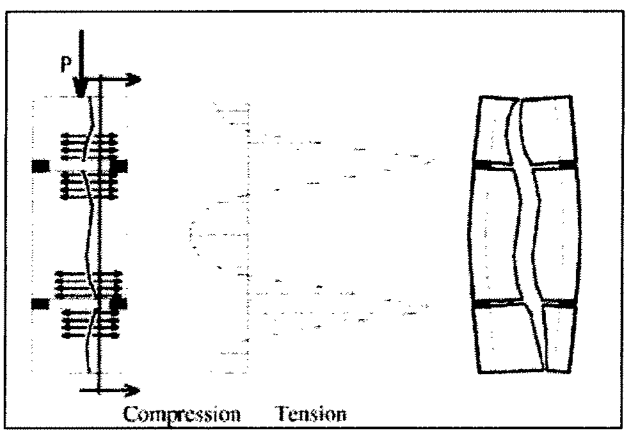


Figure 4: CMU block, vertical web splitting for compressive loading (Hamid, 2005)

The reasoning behind this is explained by Drysdale et al (1999). The mortar bed expands laterally under compression. The rate of this expansion is greater than the expansion of the adjacent CMU block webs, therefore creating a tension in these webs. Since the FRP laminate section is not in contact with the webs, they are left vulnerable to tensile failure.

Both the reinforced and unreinforced 0° and 90° specimens failed by vertical web splitting. As mentioned earlier, the load on the 60° assemblage was distributed along the horizontal mortar bed joints (the joints parallel to the long dimension of the CMU block) as a shear stress and also along the plane normal to the horizontal mortar bed joint as a compressive stress. In the unreinforced control specimen, there was a combination shear and compression failure since it also showed compressive-type failure as well as cracking along the mortar joints. The reinforced 60° specimen, though, failed completely in compression through vertical splitting.

Figure 5 describes the changes in strength and Young's modulus respectively for all of the assemblages (except for diagonal tension and joint shear).

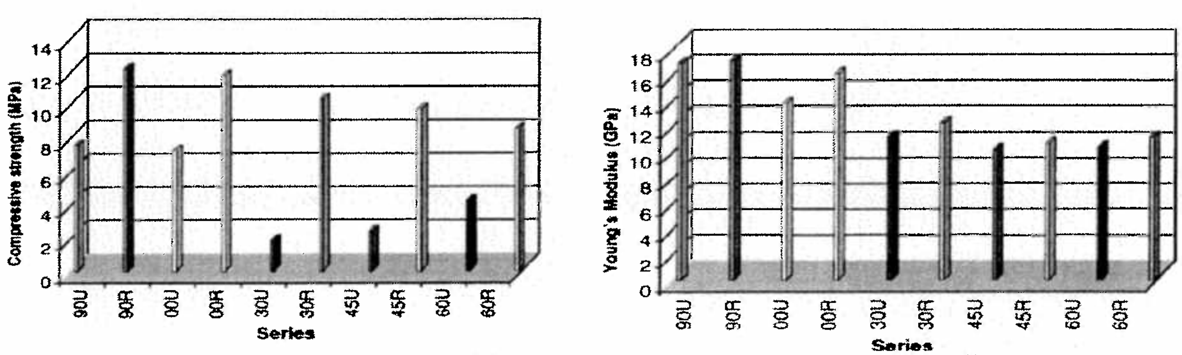


Figure 5: CMU block strength and Young's modulus improvements. The suffix U means unreinforced and R means reinforced w/ FRPs (Hamid, 2005)

In terms of applied compressive loads, the FRP retrofitted assemblages showed an overall increase in the compressive strength in comparison to the control assemblages. As displayed in Figure 5, the 0° (00R) and 90° (90R) loading configurations showed a strength increase of about 1.5 times over the control assemblages (00U and 90U). For the 60° assemblage, the strength increase was about 2 times since the strengthening was not purely compression but also shear, where the affect of FRP reinforcing is greater. Of all of the loading configurations, the assemblages having internal compressive forces as the limiting failure stress (0°, 90°, and 60°) showed the least amount of strength improvement.

The Young's modulus of the reinforced assemblages showed very little change from the control specimens.

30° and 45° Assemblages (Shear Loading)

The 30° and 45° assemblages are similar to the 60° assemblage in that the applied loading is distributed in two directions, one along the bed joints (the joints parallel to the long dimension of the CMU block), and one perpendicular to the bed joints. The detail that separates these from the 60° assemblage is the fact that the predominant stress is shear due to the steep loading angles.

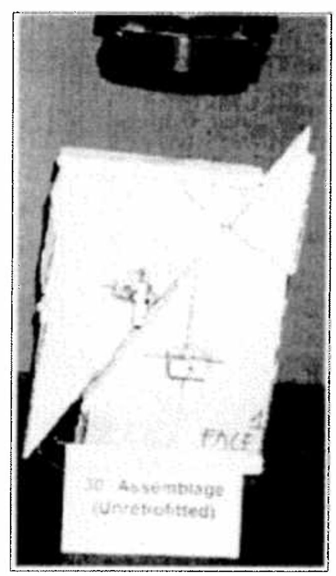


Figure 6: Shear failure along bed joint (Hamid, 2005)

The failure mode for the 30° and 45° assemblages was shear failure along the bed joint as shown in Figure 6, again due to the predominant shear stress. This failure mode occurred for the unreinforced assemblages as well as the FRP reinforced assemblages.

The strength increase between the control assemblages and the FRP assemblages were significant as shown in Figure 5. The FRP reinforced 30° and 45° specimens exhibited an increase in strength of approximately 5 and 4 times respectively.

Once again, the Young's modulus of the FRP specimens showed insignificant improvement over the unreinforced control specimens.

Joint Shear Assemblages

The Joint Shear assemblages are the other extreme of the on axis loading, where the applied loads are contributing 100 percent to shear failure. The test configuration for the Joint Shear assemblage is show in Figure 7 below. It should

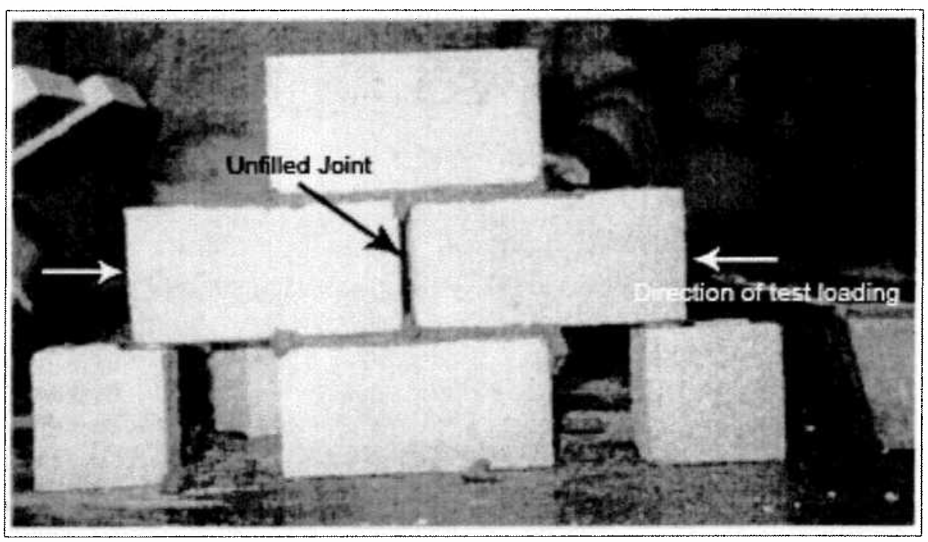


Figure 7: Joint Shear Test setup (Hamid, 2005)

be noted that the center head joint remains unfilled to ensure that the specimen will fail in shear along the bed joints.

The strength characteristics of the FRP specimens for the joint shear test exhibited tremendous improvements over the unreinforced control specimens. The FRP assemblages were approximately 8 times stronger than the control group in terms of joint shear strength, which was the biggest improvement of all the tests performed.

The failure mode for this test was of course shear failure along the bed joint, due to the mortar gap in the joint displayed in Figure 7.

Overall Strengthening Effects of FRP on CMU Block Assemblages

The results of test performed in Hamid's study showed that FRP reinforcement of CMU block structures affords significant strengthening when subjected to in-plane loading. Furthermore, it showed that *FRP strengthening had the greatest effect with shear stresses* over compressive. A clear strength continuum was observed based on the assemblage loading angle. Extreme compression was present during on axis loading. As the load axis moved away from the on-axis configuration and to an angular configuration (changing from compression to shearing stress), the strengthening effect

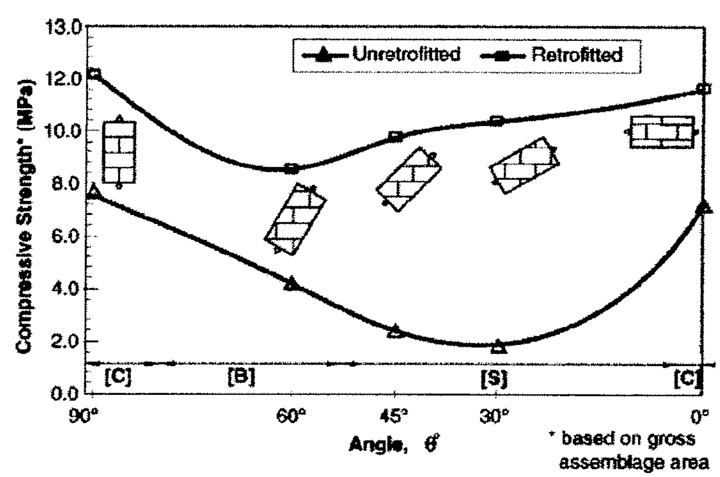


Figure 8: Load angle vs. strengthening effect (Hamid, 2005)

increased when compared to the control specimens. This relationship between the strengthening characteristics of FRPs on CMU assemblages and the angle at which the load is applied is displayed in Figure 8. It shows that the greatest on/off axis strengthening occurred with the 30° test, which induced the greatest shear component (although not 100% shear).

This determination is further reinforced by the results of the Joint Shear test, where all of the applied loading was in shear. Consistent with the theory, this test showed the greatest strengthening of the entire study since 100% of the force was shear.

FRP Strengthening on Reinforced Concrete Beams (Tensile)

A basic concept of concrete is that it is very strong in compression, making extremely effective for pavements applications. Conversely, concrete's strength is very poor in tension. Reinforced concrete beams are one of the main structural elements of construction. When loaded, there is a flexural stress that exists which is composed of a compression on the top of the beam (or the portion closest to the loading) and tension on the bottom of the beam (or the portion furthest from the loading). A major downside to the reinforced concrete structures is corrosion of the reinforcing steel (Matco, 1999), thus degrading its value. There have been many studies conducted (Grace et al., 1999; Okeil et al., 1997; Xiong et al., 2004) on the effects of subsequent strengthening of reinforced beams in their tensile sections with FRPs, as they have outstanding resistance to corrosion.

This portion of the paper will look into the tensile strengthening characteristics of FRPs on reinforced concrete beams. It is modeled off of the 2004 study conducted by Xiong et al. This study looked into the strengthening effects of FRPs on reinforced concrete at their tension region. Furthermore, it experimented with the type of FRP reinforcement material used and how that affected the strengthening properties.

<u>Carbon FRP (CFRP) Strengthening</u>

First, let us look into the basic effects of strengthening a beam with FRPs. In Xiong, two control beams were used along with 2 beams strengthened with

CFRP sheets bonded from sided to side along the bottom of the beam. One of the CFRP beams had one sheet, and the other had 2 CFRP sheets. The FRP sheets were applied using the conventional FRP fiber application with epoxy. For one of the control beams and both of the Carbon FRP beams, the tensile steel reinforcement ratio was .76. The second control beam it had a reinforcement ratio of 1.10. The beams were simply supported under four point

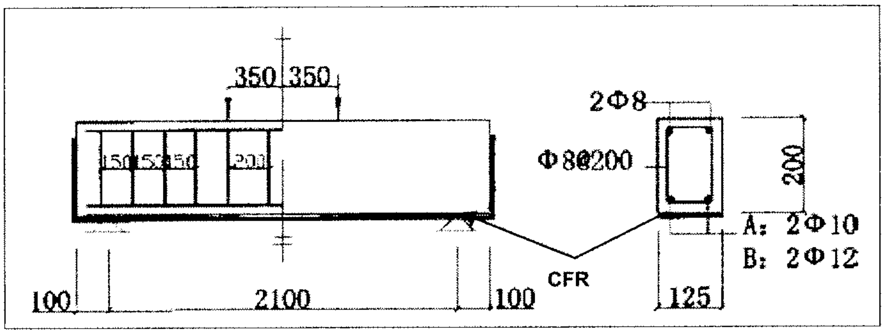


Figure 9: CFRP Concrete Reinforced Test Beam (Xiong, 2004)

loading. Figure 9 illustrates the test setup for a FRP strengthened beam.

Findings

The use of the CFRP showed a significant increase in the strength of the reinforced concrete beam of the same steel reinforcement ratio. The control beam had a yield strength of 29.38 kN, the single sheet CFRP beam yielded at 40.7 kN (39% increase over the control beam), and the double sheet CFRP beam yielded at 54.91 kN (87% increase over the control beam). The control beam with a reinforcement ratio of 1.10 had a yield-strength of 58.30 kN, very similar to that of the double sheet CFRP beam.

The ultimate strength of the beam is not the only important takeaway from this experiment though. Although the ultimate strength was greatly improved with the use of CFRPs, there was a considerable degradation of ductility in the CFRP beams. The beams that were reinforced with the CFRP showed a tremendous drop in strength following the achievement of ultimate strength. Figure 10 display this immediate drop in strength.

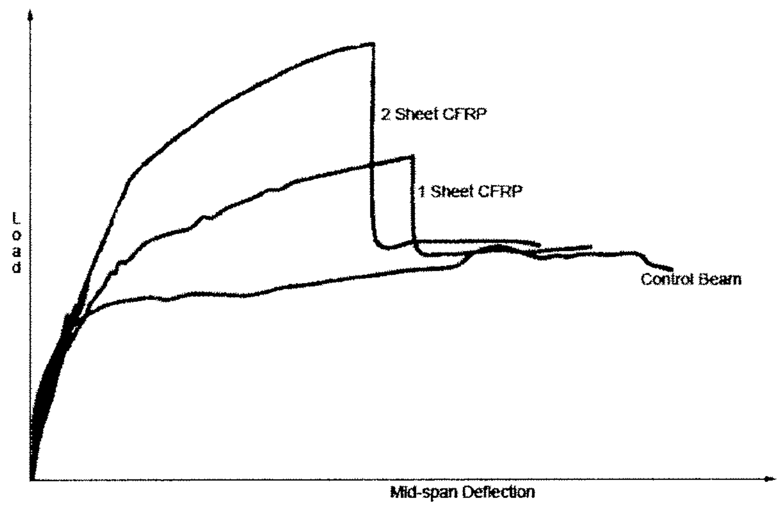


Figure 10: CFRP Ductility Issue (Xiong, 2004)

Carbon FRPs are known to have properties that of high strength and high modulus of elasticity (ACI, 2000), which translates to very low ductility or elongation at fracture. This coincides identically to its performance during Xiong's study.

Other FRP materials, although, have different elastic properties. Glass FRPs (GFRP) is one of them. While GFRPs have a lower strength than CFRPs, they are less stiff and more ductile (ACI, 2000). In order to examine the possibilities of combining the strengths of the CFRP and GFRP, Xiong performed identical testing on beams reinforced with hybrid FRPs.

Hybrid FRP (Glass and Carbon) Strengthening Effects

The hybrid FRP (HFRP) was made by applying one 100mm wide layer of CFRP to the concrete beam, followed by the application of one 125mm layer of the GFRP on top of that. Two HFRPs were constructed and tested.

Findings

The HFRP showed very similar yield-strength (slightly higher at 59.21kN) to that of the double sheet CFRP. Furthermore, the hybrid effect on the performance was substantial, successfully increasing the ductility of the beam as shown in Figure 11 below.

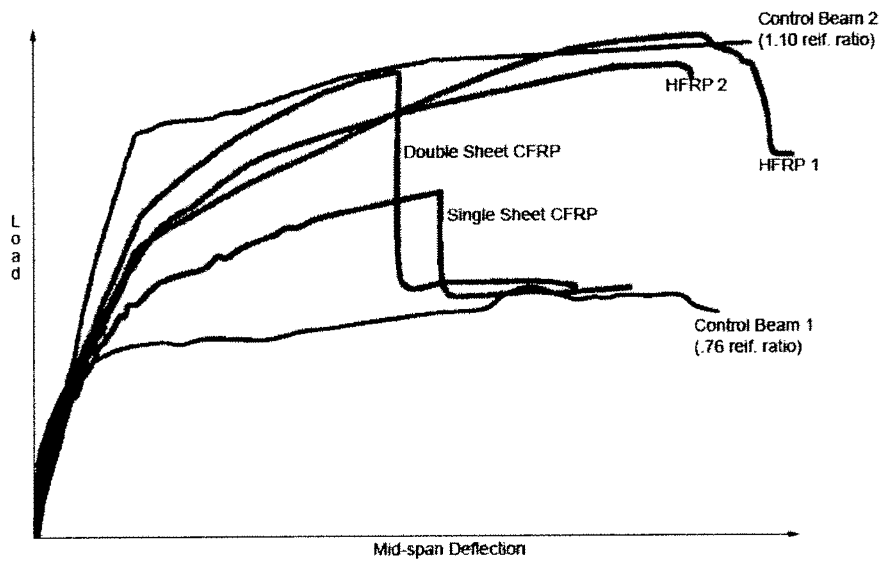


Figure 11: Hybrid (Carbon and Glass) FRP Ductility Improvement (Xiong,

As the test load increases, and as the more rigid individual carbon fibers fracture, the more ductile glass fibers will bear the load. This will delay the further fracture of more individual carbon fibers, thus increasing the overall ductility of the hybrid unit. (Qiao, 1997)

Overall Strengthening Effects of FRPs on Rein. Concrete Beams (tensile)

We have learned a few key things about FRPs' strengthening behavior in concrete beams at the tension elements. The use of only double layer CFRP will increase the strength of the beam by almost 90%. Although this increase is realized, there was a profound decrease in ductility observed in the beam, which was a predicted behavior given the characteristics of carbon fibers. Since glass fibers are known to have the reverse characteristics in terms of ductility, it was discovered that the use of the carbon and glass fiber as a hybrid FRP brings out the best characteristics in both materials: the highly ductile behavior of the glass fiber through the point of failure, and the high strength of the carbon fiber. In actuality, the overall strength of the hybrid beam was slightly higher than the double CFRP beam, which is approximately double the control beam strength. The rigidity of the beams did not change much with the use of the FRPs, as the slopes of the beams' stress strain curves in the elastic region were very similar. (Xiong, 2004)

FRP Strengthening on Reinforced Concrete Beams (Shear)

The previous section of this paper mentioned the need to investigate strengthening techniques in reinforced concrete beams due to corrosion and strength deterioration of reinforcing steel in concrete beams. Since that section focused on the strengthening of tensile elements, it is only appropriate that this section focus on the other strengthening area that reinforcing steel is used for, shear.

Experimental Shear Strengthening Factors

The FRP strengthening of reinforced concrete beams in shear is accomplished by bonding the FRP sheets to the side faces of the concrete beam. Studies that analyzed this FRP reinforcing technique include Zhishen et al. (2006); Chajes et al (1995); Sato et al. (1996); Norris et al. (1997); and (Adhikary) et al (2004). Several different factors were examined in these studies with respect to the FRP application configurations. These experimental configurations include:

- the orientation of the FRP sheet's fiber alignment (vertical, horizontal, combined) with respect to the length of the beam
- single FRP sheet vs. double FRP sheet
- the height of the FRP sheet up the sides of the beam
- the use of a U-wrap configuration (FRP sheet from one side of the beam to the other continuing through the bottom face)

Figure 12 below shows the test specimens for Adhikary's study. Adhikary's study will be the reference study for this section, as it represents the most comprehensive summary the previous studies. The following sections will be based on the results of this study.

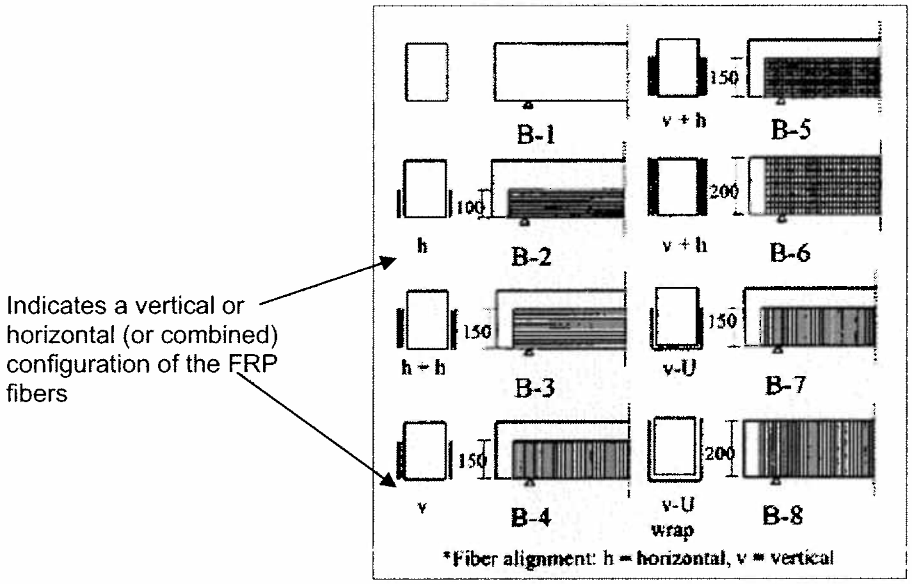


Figure 12: Concrete Beam Shear Test Specimens (Adhikary, 2004)

Figure 12 shows the specific configurations to allow for comparisons of the points mentioned above. Fiber orientation can be compared by examining beams B-2, B-4, and B-5. The effects of single vs. double FRP sheets are examined by comparing the performance of beams B-2 and B-3. The effects from the height of the FRP application on the sides are examined by comparing beams B-5 and B-6, and B-7 and B-8. And the effects of U-wrapping the beams can be looked at by comparing B-4 and B-7. For all of the beams, the FRP sheets were applied using the conventional application technique, with the use of an epoxy resin and an allowance for curing. Finally, a control beam, B-1 was tested with no FRP reinforcement.

All of the beams had steel reinforcement in their tensile regions. To ensure that the beams fail in shear, no steel stirrups were used for internal shear reinforcement.

Performance due to Fiber Alignment

The beams that were bonded horizontally were observed to fail with the rupture of the FRP fibers, whereas the vertical bonded beam, B-4, showed a failure in debonding of the FRP from the concrete. This debonding was caused by the crushing and splitting of the concrete behind the sheets. This displays increased rupture strength of vertical fibers over horizontal fibers. The shear strength of the vertical, horizontal and combined (vertical and horizontal) fibers appeared to be very similar, with strengths of beams B-3, B-4, and B-5 around averaging to about 61 KN, approximately 56% higher than the control beam.

Performance due to the Number of FRP Sheets

It appears that the addition of a second FRP sheet does not provide a significant increase in the shear strength of the beam. In looking at beams B-4 and B-5, there is only a 3% increase in the shear strength with the addition of the 2nd horizontal sheet. Although, in looking at the failure mode of the beams, one can argue this portion of the experiment to be inconclusive. Debonding of the FRP sheets due to concrete crushing and splitting behind the FRP was the case in the beams used for this comparison. Since second FRP layer does not contribute any additional anchorage strength at the FRP concrete interface, it is expected that very little additional strength will be realized. It would interesting to examine the effects of a second FRP layer on a specimen that did not fail with debonding characteristics, such as in beams B-6 and B-8. This additional iteration of the experiment would make the conclusion of multiple layering effectiveness (or non-effectiveness) stronger.

Performance due to the Height of the FRP Reinforcement

Based on the overall results of Adhikary's study, the variance of the height of FRP reinforcement up the side of the beams seemed to be the most sensitive factor in increasing the shear strength. In looking at beams B-5 and B-6, and beams B-7 and B-8, there was a considerable increase in shear strength due to the increase in reinforcement height. A 34% increase was observed from B-5 to B-6, and a 25% increase was observed from B-7 to B-8, in which both comparisons involved the increase in side FRP reinforcement height from 150mm to 200mm.

Performance due to a U-wrapping Configuration

The maximum performance of all of the beams came from B-8, which had a U-wrap configuration. It had a shear strength of 85.8 kN, an increase of 119% over the control beam. In comparing beams B-4 and B-7, both of which had a side reinforcement height of 150mm (although B-7 was a U-wrap beam), there was an increase in shear strength from B-4 to B-7 of 17%.

Stiffness Effects of Shear Reinforcement

The results of the shear stress to mid-span strain curve indicates that reinforcing concrete beams in shear has very little effect on the stiffness of the beam. Figure 13 (Adhikary, 2004) shows the nearly parallel stress-strain slopes (elastic modulus) of the FRP-reinforced beams in comparison to the control beam.

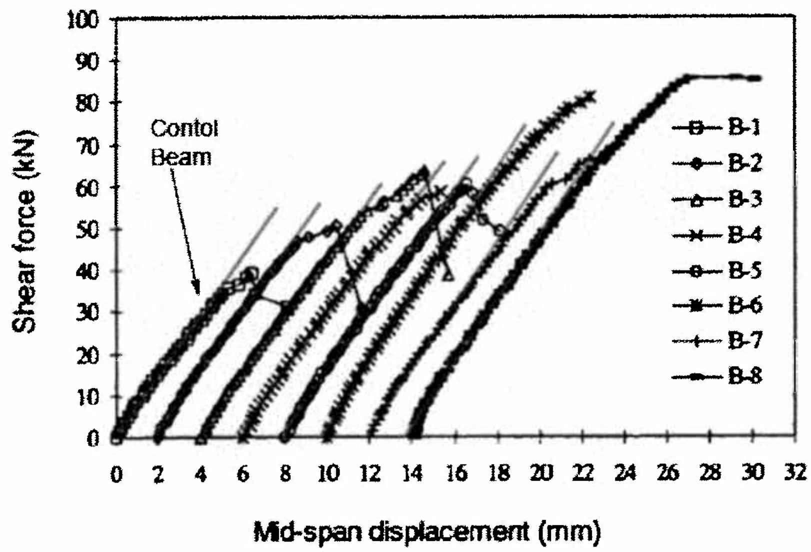


Figure 13: Stiffness effects of shear reinforcement (Adhikary, 2004)

Overall Strengthening Effects of FRPs on Rein. Concrete Beams (shear)

The use of FRP reinforcement on concrete beams for the purpose of shear strengthening provides significant results. Improvements of over 100% of an unreinforced concrete beam (in shear) can be reached with the use of FRPs. The manner in which the FRP is applied to the beam makes an immense difference on the degree of strengthening achieved.

The use of multiple layers of FRP on sides of the beam tended to have minimal effects on the shear strength. An issue that came up was debonding of the FRP plate from the concrete beam. This was the case with the beams where multiple layer effects were examined, and the application of the 2nd layer provided no improvement on the bonding characteristics.

The orientation of the FRP fibers did have an effect on the failure mechanism and showed that vertically aligned FRP sheets provided a higher fiber rupture strength. Nevertheless, the overall shear strengths of the beams, regardless of the orientation (vertical, horizontal, combination) seemed to be very similar. Again, if an additional iteration of the test was conducted on beams that did not debond, the combination beam and vertically aligned beam (which failed partly from debonding) may have yielded different results.

The use of the U-wrap technique showed considerable improvements over the side-only reinforced equivalent beam. Although, the configuration variable that showed the most efficient improvement was the height of the reinforcement on the sides of the beam. The highest jumps in shear strength were as a result of increasing the reinforcement height.

The beam with the largest strength improvement over the control beam was beam B-8. This beam utilized the asset from the 2 most sensitive variables, height of reinforcement and U-wrap technique, and applied it to one beam.

FRP Strengthening Using Mechanically Attached FRPs

All of the FRP strengthening experiments discussed thus far have utilized the conventional method for applying the FRP strips. This method, as explained in the introductory section to FRPs, applies the FRP sheets to the construction material with a type of resin. Almost all experiments encountered in the references to this paper used this conventional application method. While this method is effective for many uses, there are also instances where it is not practical.

Time Issues with Conventional Application

The main issues with the conventional application method are with regards to time. Not discussed in the earlier experimental sections that used the conventional application method was the procedures and time required to construct the test specimens.

The time to complete an FRP application using epoxy bonding is described in terms of days. Several measures must be taken to ensure that the maximum possible bond is achieved. Important to the procedure is the preparation of the surface that the FRP is going to be bonded to. This typically entails sandblasting, grinding, cleaning, and smoothing of the surface to ensure its suitability for bonding. Furthermore, the epoxy system must be mixed with great precision and applied carefully to produce a good bond line. Once applied, there is a time, typically ~24 hours (Sika, 1999), that the strip must not be disturbed for. Upon completion of the application, the system can take between 4 to 7 days for the design strength to be achieved (Masterbuilders, 1998). In the majority of the FRP strengthening studies that have been completed, specimens were constructed using the conventional epoxy bonding method described above. These specimens took about 4 hours to apply and 5 days to cure.

Though these time requirements may be acceptable for most applications, there are some that this is not acceptable. In the civilian sector, there is a need to repair highways and bridges in an expedited manner in order to reduce the amount of traffic congestion and frustration caused by repair.

Also, there is an enormous requirement in the military for quick structural strengthening. This requirement is frequently driven by time critical missions. Often times in foreign settings, existing infrastructures are not adequate to support the throughput of military equipment and materials. As a result, speedy strengthening of these structures (mostly bridges) is necessary, so as not to delay the mission.

Specialized Personnel Requirements with Conventional Application

Another issue associated with the conventional application method is the requirement for specialized personnel in order to accomplish effective application of the FRP. Again, in a military setting, the availability of these specialized personnel is low. Ideally, the process can be one where large masses of people can be trained to apply the FRPs.

Power-Actuated Nails for Mechanical Fastening

Several studies looking at alternate, speedier ways to apply the FRP strips to building materials, and examining their performance have been conducted. To date, one of the most popular methods is to mechanically attach the RFP strips using what is essentially a nail gun and nailing the strips to the concrete. Although this method sounds somewhat crude and trivial, it is actually somewhat complicated on a microscopic level. The nails used are not typical nails, but powder-actuated fasteners. Basically, the penetration of the fastener into the concrete generates friction and heat. This heat that is generated causes sintering and a chemical bond is created between the nail and the concrete. (CEB, 1994)

This section will reference two studies by James Ray (2003) and Anthony Lamanna (2004) in examining the difference in strengthening capabilities on reinforced concrete beams when using mechanically attached FRP strips vs. using conventionally applied FRP.

Experimental Setup and Variables

The studies mentioned above used very similar procedures and variables in their testing. The basic question that the author's were trying to answer was: how different are the strengthening capabilities between the beams using the FRP strips applied via the *conventional method* and the beams with the FRP strips applied with the *mechanical method*? In addition to this, there were other ancillary behaviors that were tested.

While there have been other previous experiments testing these same issues (mechanical attachment of the FRP strips), the majority of the tests were conducted on beams and loads that were scaled down for economic reasons. Since the sized of the fasteners used for the scaled down beams and the full scale beams were the same, Ray wanted to look into the effects of the fastener's scale relationship to the beams. *Full and small scale beams* were tested in his study.

Another factor that was examined in Lamanna and realized in Ray was the change in performance due to *pre-drilling* of the mechanical fasteners vs. those that were simply driven into the concrete.

To determine whether the *widths of the FRP strips* played a significant role in the strengthen capabilities of the mechanically attached beams; width was also used as a variable in Lamanna.

Since the only connections between the beams and the FRP strips for the mechanically connected specimens were the powder actuated fasteners, Lamanna also looked into the effects of *fastener spacing* (especially in the moment span of the beam) and *fastener depth*.

All of the beams that were tested contained the same flexural and shear steel reinforcement. Both studies included specimens of different concrete strengths in addition to the factors mentioned above.

Experimental Results

The main point of these experiments was to determine whether the use of mechanically attached FRP strips can produce comparable performance to that of the conventionally bonded strips. The results of these experiments show that this is in fact the case. Figure 14 displays the performance of the 2 beams as

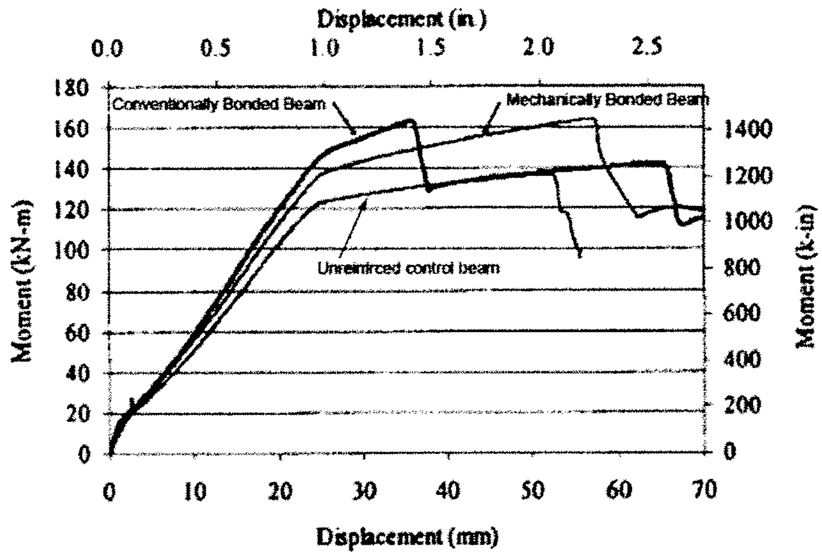


Figure 14: Mechanically vs. Conventionally Bonded FRP Strengths (Lamanna, 2004)

well as the control beam with no FRP. The two important things to note from this figure are the significant strength improvement of the mechanically attached beam from control beam, and also the similar yield and ultimate strengths of the mechanically and conventionally attached beams.

In addition to the strength characteristics of the beams, the experiments showed that there the mechanically attached beams gave more favorable results than the bonded beam in terms of ductility. As shown on Figure 14, the conventionally bonded beam does not provide much displacement between the yield point and the point where ultimate strength is reached. At that instance, there is an abrupt drop in strength which corresponded to the delamination of the FRP strip. The mechanically bonded beam, which yielded at strength 6% lower and had an ultimate strength virtually equal to the conventionally bonded beam, displayed considerably greater ductility with three times more displacement from the time of yielding to the time of ultimate failure.

Beam Scaling Effects

The results of the Ray's tests did show that there is an effect of beam scale to the strengthening characteristics of the mechanically attached FRP strips. The small scale FRP beams showed greater strengthening effects than their full scale counterparts. The small scale FRP beams had about a 20% increase in yield strength and a 30% increase in ultimate strength form the small scale control beam, whereas the full scale FRP beams had about a 13% increase in yield strength and an 18% increase in ultimate strength from the full scale control beam. This scaling effect is mainly due to the increased relationship of the fastener depths in relation to the beam depth.

A further experiment would be more conclusive if as the beams size increases, so does the nail sized (maintain fastener depth to beam depth ratio). One might predict that a more linear size-to-strength relationship would occur with this experiment than what was realized in Ray's tests.

Effects of FRP Strip Width

The experiment that examined the effects of the width strips yielded interesting results that were critical in understanding the drawbacks of using mechanically fastened beams. This test looked at a beam with two, four-inch strips (eight inches total width) versus a beam with only one four-inch FRP strip attached. Figure 15 shows the results of this test. The 8 inch width specimen showed higher yield strength although the 4 inch width specimen had a higher ultimate strength. It turns out that an important sub-factor when looking at strip width is the *edge distance*. The 8 inch specimen has a smaller edge distance than the 4 inch, which led to significant initial cracking upon driving of the fastener. This cracking has proven to play an important role on the performance of the mechanically attached FRP beams, in that is causes early detachment of the FRP strips. The increase in yield strength of the 8" beam shows the strengthening of the additional FRP width. After the beam has yielded, the ultimate strength is reached followed by two drops in strength. These two drops correspond to the detachment of the two strips, thus not allowing the strips to reach their true ultimate performance. As a result, the true effects of widening the strip section were not determined. The prevention of this cracking upon driving the fasteners will be discussed in a later section.

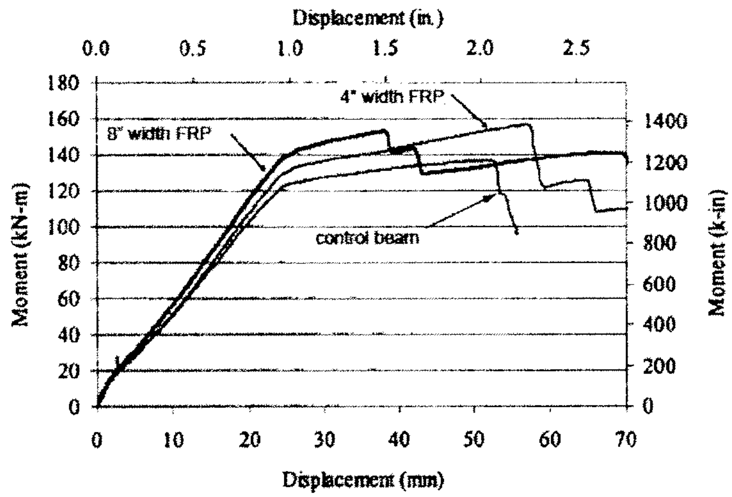


Figure 15: FRP width effect for mechanically attached FRP strips (Lamanna, 2004)

Fastener Spacing

In all of the beams for both Lamanna and Ray, the fasteners were spaced at 2 inches. Although, Lamanna conducted a test to determine if the increase of fastener spacing in the moment span of the beam had any performance consequences. A mechanically attached FRP beam was constructed with 8 inch fastener spacing in the center span (moment span) of the beam and 2 inch spacing for the rest of the beam as shown in Figure 16. Results of the

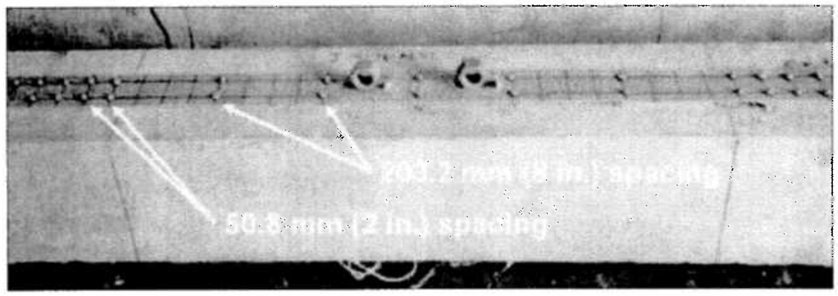


Figure 16: Reduced fasteners in moment span (Lamanna, 2004)

test showed that the elastic modulus and yield strength of the beam were virtually identical to the beam with consistent 2 inch spacing throughout. The difference was in the ductility and the ultimate strength. The 8 inch moment span beam had an ultimate strength 6% lower than the normally spaced beam, and 50% less deflection than the normally spaced beam between yielding and ultimate failure.

In addition to this, a test was conducted on a beam that had 3 inch spacing throughout, vice the typical 2 inches of the rest of the test specimen. The results of this test were similar to that of the increased spacing in the moment span. The performance of the 3 inch spaced beam in the elastic range (elastic modulus, yield strength) was almost identical to the 2 inch spaced beam. The ultimate strength of the 3 inch spaced beam did show a 3% decrease in ultimate strength when compared to the 2 inch spaced beam.

Effects of Concrete Strength

As mentioned earlier, Ray's study included testing on beams with identical reinforcing configurations but with different concrete strength properties. The beams had concrete strengths of 21 and 42 MPa. The results of the test indicated that the beams with the lower concrete strength showed a greater strength improvement over the control beam than the high concrete strength beam in terms of ultimate strength. Again, this was found to be attributed to initial cracking upon nailing the fasteners to the beam. Since the higher strength concrete was more brittle, the nailing of the fasteners inflicted more damage (cracking) to the high strength beam.

Predrilling of the Fasteners

The tests have determined that the impact of the power actuated fasteners during nailing causes initial cracking in the concrete. Figure 17 is an illustration that shows the cracks that formed from nailing fasteners with little edge distance.

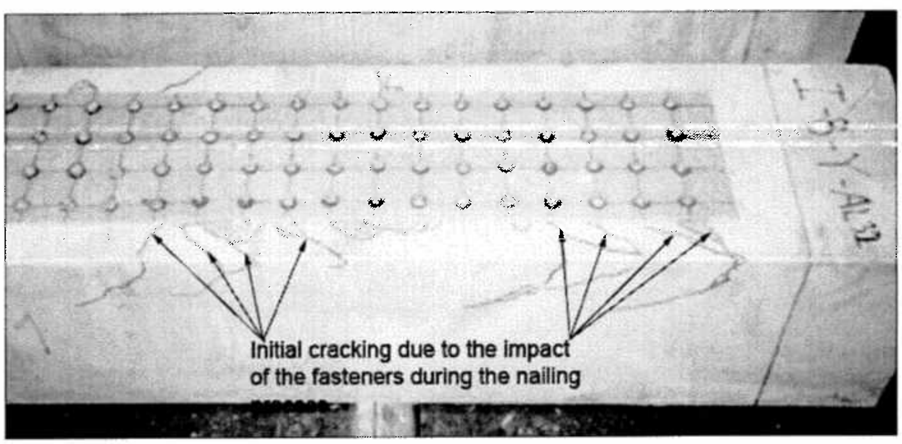


Figure 17: Initial cracking due to nailing (Lamanna, 2004)

To mitigate this cracking effect, Lamanna tested beams with predrilled holes prior to nailing the powered actuated fasteners. The performance of these beams was compared to the similarly configured non-predrilled beams to determine the effects of predrilling.

The results of the test showed that predrilling was an effective way to handle the problem of initial cracking. The amount of initial cracking due to the nailing of the fasteners was significantly reduced. As far the beam performance, the predrilled and non-predrilled beams behaved almost identically in the elastic range. Even after yielding, the beams followed the same stress-strain curve. The difference was, as the non-drilled beams reached their ultimate strength, the predrilled beams continued to achieve a 5% greater ultimate strength and twice the post-yield displacement. Repeat tests of the widened beam strip and the higher concrete strength using predrilled fasteners would produce results that show a more accurate depiction of the strength effects since the strips would stay connected long (at higher strengths).

The use of a longer fastener on another predrilled beam was examined in the same test as the one discussed above. Interestingly enough, this beam also followed the identical elastic behavior as the other 2 beams (non-predrilled and predrilled). This beam yielded at the same strength and then continued on an identical path as the other 2 beams. As mentioned earlier, the non-drilled beam failed first, then the pre-drilled beam failed in an extension of the same stress/strain slope, and finally, the lengthened pre-drilled beams failed, again on the same slope and with increase ductility (about twice the displacement of the shorter predrilled beam).

Overall Effects of Mechanically Attached FRP Strips

The use of mechanically attached FRP strips has shown to be worthy of use for the specialized scenarios where conventional application is unfeasible. In general, it provided similar strengthening behavior to the conventionally bonded beam (slightly less yield strength and equal ultimate strength) with far more rapid and less technical installation. It also improved the behavior over the conventional application in terms of ductility.

The studies also showed that there are other factors that affect the degree at which the mechanical application method can strengthen the beam. An important observation with regards to previous tests was made in realizing the different in performance between small scale beams and full scale beams. It is now known that this is not a linear comparison, and the strength of the beam can not be increased at the same rate as the size is increased. There is some strength degradation factor that must be taken into account when up-scaling the beam, mainly due to the relative depth of the fastener and the beam depth (assuming the same fasteners for the small and large scale beams). Another scaling experiment which includes the scaling of the fasteners to maintain the fastener depth/ beam depth ratio would be a valuable experiment in confirming this fastener depth effect.

It has also found that the edge distance, the distance between the edge of the FRP strip and the edge of the beam, plays a part in the degrading the ultimate moment due to the initial cracking upon nailing of the fasteners. This initial cracking does not allow the FRP strip to reach its full strengthening potential since premature separation of the strip occurs. Although this initial cracking was experienced in all beams when nailing the fasteners, increased cracking (thus increased ultimate strength degradation) was experienced in the wide strip beams and also the beams with high concrete strength.

To combat the cracking effects that occurred with each of the test beams (more prevalent in some as mentioned above), Lamanna looked at predrilling holes prior to nailing the fasteners. He found that this was very helpful in reducing the initial cracking and allowed the beam to fail in a form other that the detachment of the strip.

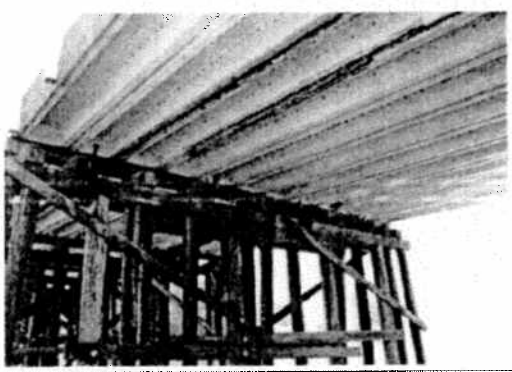
Further test with all beams predrilled would produce more untainted results about strength effects since failure would most likely be dictated from the strength attribute of the test and not the detachment of the strips.

Of all the tests, the mechanically attached beam with the highest ultimate strength was the beam with the single 4 inch strip, fastened at 2 inch spacing, using the longest fasteners (greatest fastener depth to beam depth ratio), and with predrilling prior to nailing the fasteners. This beams had an ultimate strength equal to the bonded beam of equal strip width.

The Use of FRPs on Actual Dilapidated Bridge Structures

The experiments and data above were all conducted using test beams constructed under the control of the experimenter. This section will look at a study conducted by Sherrill Ross et al. (2004), which applied similar methodology to that used in the studies above, although the test beams were actual beams from the Neil Bridge in Vancouver Island, British Columbia

The Neil Bridge was made of reinforced concrete beams. It was constructed between 1956 and 1960. The beams that made up the bridge deck were



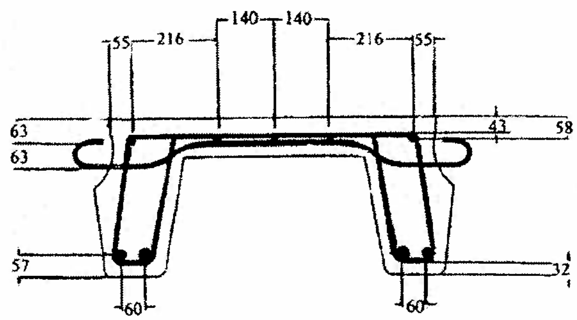


Figure 18: Neil Bridge Channel Beams. Left photo shows the channel beams underneath the actual bridge deck. Right photo shows the cross section of the channel beam with typical steel reinforcement. (Ross, 2004)

channel beams as displayed in Figure 18. These channel beams are highly deteriorated or structurally deficient (like many of the bridge in the US today).

Experimental Procedures

The procedures to Ross's study were somewhat simple. Three beams were taken from the Neil Bridge. One of the beams was intended to be tested with no reinforcement as a control beam. The other two beams were reinforced with 2 different methods FRP strengthening. Both of the FRP beams used Glass FRP (GFRP), although one was installed by the conventional sheet fabric method, and the other one was using the a GFRP spray application method. The GFRP reinforcing was placed on the bottom side of the channel beams as shown in Figure 19.

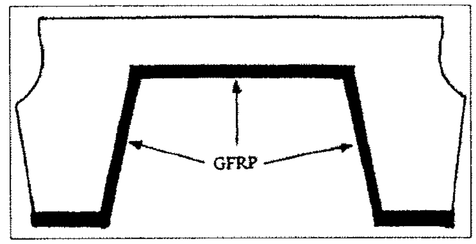


Figure 19: GFRP Placement on Channel Beams. (Ross, 2004)

It is very important to note that all three of the beams had different degrees of deterioration and exposed steel reinforcement. Beam one (the control beam with no FRP reinforcement) appeared to have the most physical deterioration of all three of the beams, with the top of the beam displaying significant damage. In a sense, this disregards it as a true "control beam." For a more accurate demonstration of the GFRP effects, prior to applying the GFRP to the 2 beams, all three of the beams were loaded in their elastic range in order to determine their initial stiffness. This way, the reinforced beams will have a stiffness comparison of the same beams with and without the FRP.

The stress strain behavior was recorded for all the beams in testing to failure.

Overall Results

The application of GFRP to the channel beams of the Neil Bridge showed noteworthy improvements in stiffness and ultimate strength. Figure 20 shows the

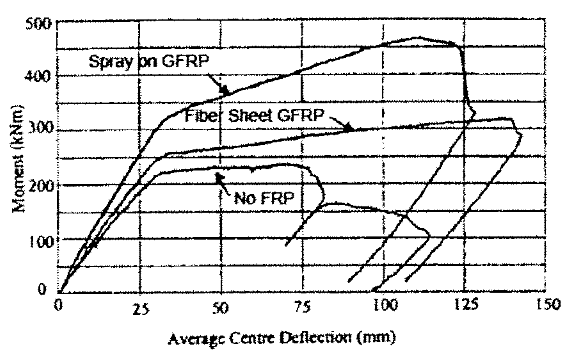


Figure 20: Neil Bridge Stress/Strain Curve. (Ross, 2004).

results of the test with the beam's stress/strain curves. Again, it is important to note that all of the beams had different levels of deterioration prior to testing. Therefore, a direct comparison of the spray-on application method to the fiber sheet method would be somewhat inconclusive. However, the general trend between the no-FRP beam and the other two beams shows an increase in strength with the GFRP, regardless of the application method. Furthermore, the tests conducted to all the beams prior to an FRP reinforcement showed that after the 2 beams were reinforced, their Young's Modulus increased in the Spray-on and Fiber Sheet beams by 25% and 41% respectively. And these results were on the same beams, therefore disqualifying the issue of different deterioration on the three beams.

FRPs in Industry Today

With all of the studies and discussion about the use of FRP polymers in structures, it has definitely made an entrance as a subsection of the structural industry today. Several companies have arisen with services to provide FRP structural strengthening of facilities. Facilities like highway bridges which are in dire need for structural retrofit, although can not afford to be closed to its users due to societal need. The practices of these companies that offer FRP strengthening are in line with the conclusions and deductions of the studies mentioned within this paper.

Tyfo Fibrwrap Advanced Composite System from Fyfe Co., LLC was the first externally bonded FRP system used for structural strengthening of masonry, concrete, steel, and wooden structures. Their construction organization, Fibrwrap Construction Inc. has completed over 1000 seismic upgrades and retrofits since the early 1990's.

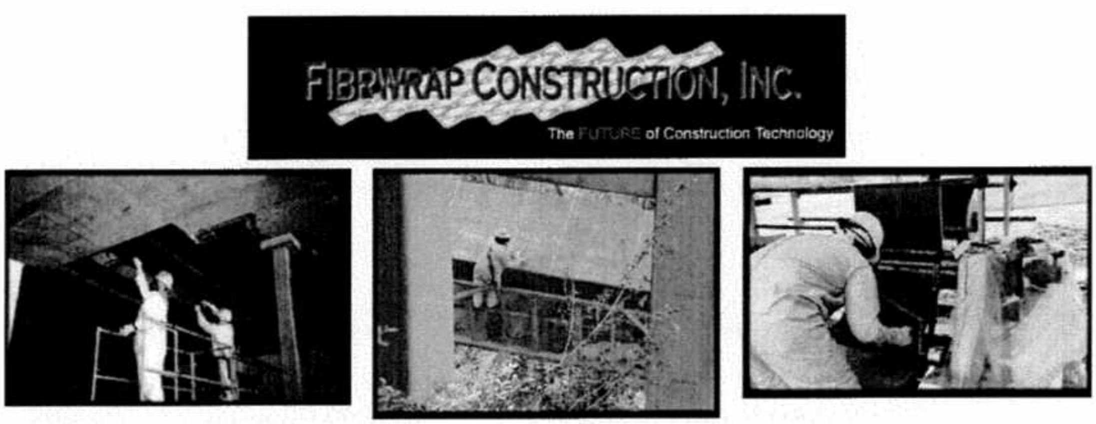


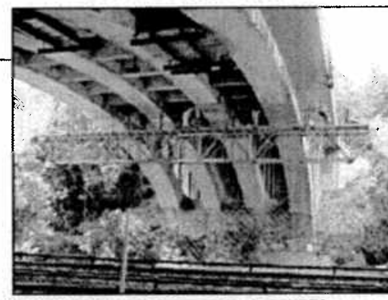
Figure 21: Fibrwrap Construction Inc., Industry pioneers for FRP strengthening

One of their projects that is consistent with the theme of this paper is Woodland Viaduct Bridge project. It represents a common problem that transportation/structural engineers face. The project description is displayed below. (Fyfe, 2006)

FYFE Co. LLC "THE FIBRWRAP® COMPANY"

Project UPDATE

Woodland Viaduct Bridge

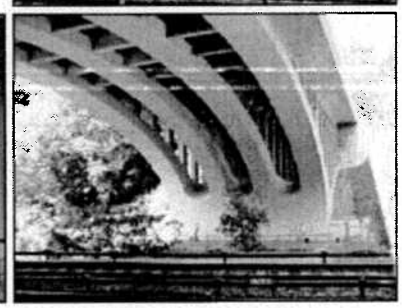


Location: Westchester County, New York

Consultants: Vollmer Associates

Contractor: Fibrwrap Construction, Inc.

Completion Date: September 2001



Description: This 75-year-old concrete, open spandrel arch bridge was not designed to meet modern day traffic requirements. Structural analysis determined that rehabilitation of the bridge would include replacement of the superstructure and strengthening of the existing arches.

Problem: Following a seismic analysis, it was determined that the arches had inadequate confinement. Fiber reinforced polymer composites (FRP) were chosen as an innovative method for strengthening. This type of repair would add the required strength without altering the appearance of the bridge.

Solution: Three layers of the Tyfo® SEH Glass Composite System were used to provide the required confinement to the arches. The design included special details to account for the irregular cross section of arches and the presence of columns. The composite system was applied in half sections around the arches, overlapping in the center. In addition, special scaffolding was used that attached to the bridge, eliminating the need for supports on the ground. This system did not interfere with the commuter railroad below the bridge and minimized the impact of construction on surrounding areas.

Benefits: Overall, the Tyfo® SEH System provided a state-of-the-art rehabilitation design for this historic bridge. This unique solution maintained the historical appearance of the bridge and had minimal impact on the community.

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